401

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DISCUSSION
OF PROCEEDINGS - SEPARATES

195, 228, 311, 315

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DIVISION

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DISCUSSION OF SPECIAL PROCEDURES FOR PAVEMENT DESIGN PROCEEDINGS-SEPARATE 195

R. HORONJEFF,[9] A. M. ASCE.—A concise presentation of the method used by the Navy for designing airfield pavements has been offered by the author. There is little that can be added except some general comments.

[9] Lecturer and Research Engr., Inst. of Transportation and Traffic Eng., Univ. of California, Berkeley, Calif.

Mr. Palmer asserts that the determination of the thicknesses of the individual components of a pavement is as important as the determination of the total pavement thickness. Most design procedures evaluate the total thickness of pavement required. The total load on a single or dual wheel is used as the criterion for design. It is important to note that, normally, the gross load on a wheel is more important than the contact pressure for determining the total thickness of pavement. However, contact pressure becomes increasingly important for the design of the layers near the surface of the pavement because of the high shearing stresses induced in the material. The stresses induced by aircraft operating during World War II did not normally exceed the shearing resistance of a well-compacted base and bituminous surfacing. Therefore, most pavement failures were attributed to failure of the subgrade material because of the inadequate total thickness required to distribute the load to the subgrade. As tire pressures increase, more attention will have to be given to the design of those components near the surface. The Navy criterion of 50,000-lb load at a pressure of 150 lb per sq in. usually governs for the determination of the total pavement thickness, but the pressure of 250 lb per sq in. associated with a wheel load of 20,000 lb is probably the governing factor in the design of the bituminous wearing surface. In this connection, Mr. Palmer states that attention is being given to the design of asphalt mixes in order that the pavements may withstand the exceedingly high pressures imposed by some jet aircraft.

The author has emphasized that the Navy method of design is not dependent on formulas but on the results of trial pavement sections. The formulas are used only to estimate the required pavement thickness for the construction of trial sections. In order to use the formulas, however, it is necessary to determine the modulus E_2 of the subgrade by plate-bearing tests; this type of test is cumbersome. In many instances, an estimate of the thickness for the construction of trial pavements might be made by utilizing design methods which do not require plate loading. Approximate correlation between CBR-values and E-values have been presented by the author and could be used for this purpose.

The Navy procedure emphasizes wheel load and corresponding contact pressure as the governing criteria for determining pavement thickness. In many areas of the United States extreme frost conditions are encountered. In these areas a blanket of non-frost-susceptible materials—to insulate completely

or partly the subgrade—is required. The thickness of such a blanket might be greater than the thickness of the pavement computed only on the basis of wheel load and tire pressure. In such cases the greater of the two thicknesses is adopted for design.

Mr. Palmer asserts that one of the advantages of the procedure used by the Navy is that it permits an evaluation of the load-distributing characteristics of various types of base courses. It is further stated that the moduli of base courses vary from 25,000 lb per sq in. to 200,000 lb per sq in. The writer has attempted to illustrate the effect of variable strength base courses on the required thickness of pavement in Fig. 8. Fig. 8 shows, for example, that, for a subgrade whose E_2-modulus is 1,000 lb per sq in., an increase in the modulus of the base course (E_1) from 50 kips to 100 kips permits a decrease in pavement thickness of 16 in. for a single-wheel load of 50,000 lb at a contact pressure of 150 lb per sq in. From this relationship several observations can be made. As the moduli of the base course increase, large differences in moduli do not produce large changes in thickness. If the E_1-moduli are increased from 100 kips to 150 kips, the thickness is reduced by approximately 6 in. compared to the 16-in. reduction when the moduli are increased from 50 kips to 100 kips.

From Fig. 8 it can be seen that the distributing characteristic of asphaltic concrete surfacing laid on bases whose E_1-moduli are between 25 kips and 100 kips might be quite important, and the substitution of base course for asphaltic concrete in the trial sections may be questionable in these instances. It would be of interest to know the range of moduli of various asphaltic mixes, if such values were available.

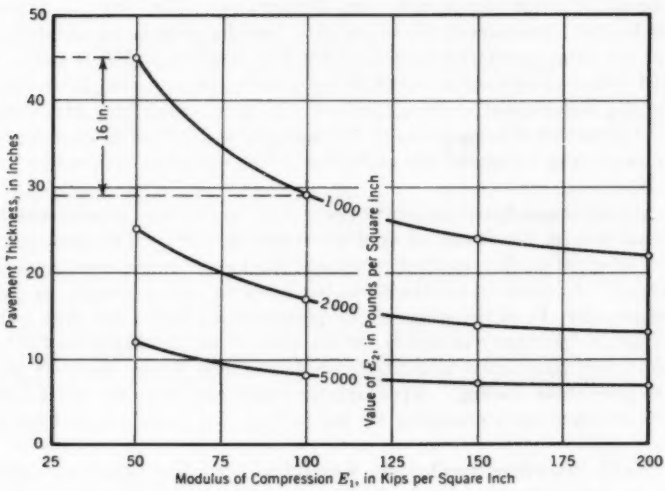


FIG. 8.—THE EFFECT OF LOAD-DISTRIBUTING CHARACTERISTICS OF BASE COURSES

L. A. PALMER.[10]—Mr. Horonjeff's discussion helps to clarify certain items.

[10] Eng. Consultant, Soil Mechanics and Paving, Bureau of Yards and Docks, U. S. Dept. of the Navy, Washington, D. C.

It is believed that Fig. 8 may tend to be somewhat misleading because the range of subgrade moduli (from 1 kip per sq in. to 5 kips per sq in.) does not represent the most unfavorable subgrade condition. Fig. 9 shows similar relationships for a very wide range of subgrade and pavement conditions. From Fig. 9 it can be seen that the total over-all pavement thickness, with E_1 equal to 200 kips per sq in., varies from 11 in. to 40 in. as E_2/E_1 varies from 1/100 to 1/500. This range of ratios represents a subgrade modulus range of from 400 lb per sq in. to 2,000 lb per sq in.—a rather normal range when fine-grained soils and high water tables are encountered.

Fig. 9 shows that maximum information is obtained when the ratio E_2/E_1 (rather than the individual values) is considered. The curves in Fig. 8 show pavement thicknesses for a constant E_2 and a variable E_1. Fig. 9 shows variations in pavement thickness for a constant E_2 and a variable (E_2/E_1)-ratio. These variations are quite pronounced for E_1 exceeding 200 kips per sq. in. These theoretical relationships have been verified by actual plate loadings.

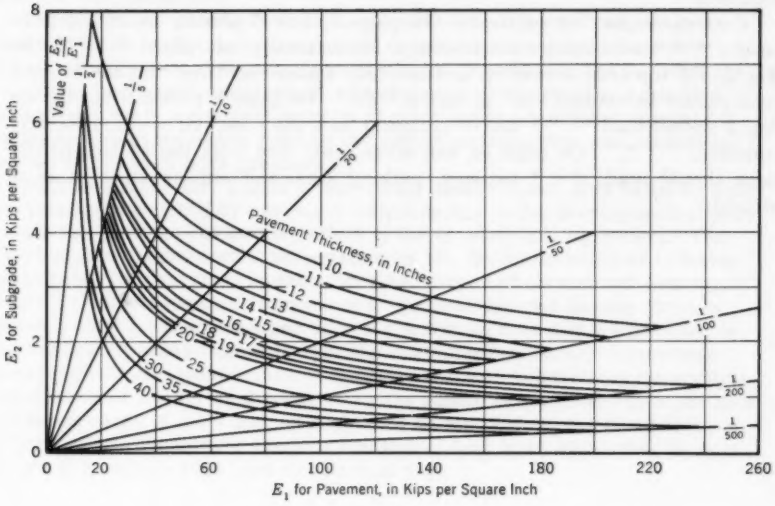


FIG. 9.—THICKNESS OF FLEXIBLE PAVEMENT FOR TIRE LOAD OF 50,000 LB AND TIRE PRESSURE OF 150 LB PER SQ IN.

There are great differences among load-spreading characteristics of different soils, subbases, and base courses. The mass of in-situ material that is affected in CBR tests is too small and too poorly representative to indicate these wide differences.

It is true that plate loading requires much time and effort, and it would be desirable, if possible, to use quicker and simpler test procedures. However, the economy realized by the use of procedures outlined by the writer has been substantial; it does not seem advisable to risk the loss of these savings until a very reliable correlation between these procedures and more simple ones has been established.

It is fortunate that granular subgrades exist at most air stations in the North Atlantic coastal areas. However, frost-heave problems have been encountered, to some extent, in these areas, and a blanket of nonfrost-susceptible materials is the only solution (as was pointed out by Mr. Horonjeff). The expression "nonfrost-susceptible materials" has not been completely and accurately defined. For example, it has been demonstrated that soils containing less than 3% of material of particle size equal to or less than 0.02 mm are nonfrost susceptible. It has not been shown, however, that all nonfrost-susceptible materials contain less than 3% of material of particle size less than 0.02 mm.

Corrections for Transactions.—On page 5, line 5 should be changed to read "* * * are being constructed. Mechanically stabilized * * *." In Fig. 2, the top scale should be entitled "E_2" instead of "E_1," and the bottom scale should be entitled "E_1" instead of "E_2." On page 9, the third line before Eq. 2 should read "* * * above optimum and the observed w-value forth is condition * * *." On page 14, the seventeenth line from the bottom of the page should read "* * * taxiway, parking area, and similar areas at each airfield."

DISCUSSION OF REDUCTION IN SOIL STRENGTH WITH INCREASE IN DENSITY. PROCEEDINGS-SEPARATE 228

CHARLES R. FOSTER,[1] A. M. ASCE. The data presented by Mr. Barber form a welcome addition to the information on the effect of different types of compaction. The differences in strength at equal moisture and density which he has found with the different types of compaction, particularly between impact and static, are in general agreement with findings at the Waterways Experiment Station. Mr. Barber's last statement that "the strength may increase or decrease with increased density depending upon the structure produced by the method of compaction" points out the fact that the statements made in the basic paper were not qualified as to type of compaction. One of the basic purposes of the paper, however, was to show the behavior that develops under modern compaction equipment and to show that similar behavior which had long been noted in samples compacted dynamically agreed with the field behavior.

The two cases cited by Mr. Barber tend to infer that the reductions in strength noted in the basic paper could have been caused by differences in structure rather than the development of pore pressures. The fact that the reduction in strength occurred at a combination of moisture and density which resulted in a constant quantity of air voids is considered conclusive evidence that in the cases cited by the author the reduction was related to the development of pore pressure.

With respect to Mr. Halton's interesting observations, it is believed that the failures described by him were definitely due to the development of pore pressure under the compactive effect of the construction equipment. The "springiness" under traffic as reported by Mr. Halton is believed to be the same behavior described by the author as "large deflections and weaving." Also, air voids computed from the average moisture and density given in Table 2 show a value of 2.4 per cent in the failure area and 8.3 per cent in the nonfailure areas indicating that the critical air voids are somewhere between these two values. There is little reason to believe that the critical value of about 3.5 per cent reported by the author is applicable to more than a limited range of soils and, as suggested by Mr. Halton, the critical value of air voids is probably a complex function of load and loading rate as well as the dimensions, type, and condition of soil.

1. Engineer, Chief of Flexible Pavement Branch, Soils Division, Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi.

DISCUSSION OF ESTIMATING THE TRUE CONSOLIDATION BEHAVIOR OF CLAY FROM LABORATORY TEST RESULTS
PROCEEDINGS-SEPARATE 311

ROBERT L. CRISP, JR.[1] J. M. ASCE.—Mr. Schmertmann's paper is a valuable contribution to the field of Soils Mechanics in that the effect of previous geological history and sample disturbance are recognized as prime factors in selecting representative compression curves from laboratory consolidation test data.

With reference to paragraphs titled "The Geological Rebound of a Preconsolidated Clay" and "Reconstruction and Use of the Estimated True Pressure - Void Ratio Diagram" of Mr. Schmertmann's paper, it has been the experience of the writer that the slope of the rebound curve of clay soils is a function of the inherent structure of the soil and the consolidation pressure from which rebound begins. Considerable variation in the slope of rebound curves has been noted in samples subjected to repeated load and unload cycles. In general, the slope of the rebound curve tends to increase with an increase in applied stress. Therefore, the rebound curve will have innumerable slopes, each a function of the maximum applied laboratory stress. Figure 1 represents plots of laboratory test data for samples subjected to a cyclic load procedure. The variation in slope is quite evident in the graphical comparison plotted with each pressure void ratio diagram.

It is believed that an arbitrary selection of the rebound curve, obtained by unloading from the maximum applied stress, to approximate geologic rebound can lead to erroneous corrections in locating Point "P", Figure 9, of Mr. Schmertmann's paper. The location of this point is critical with respect to settlement computations, since the slope of the virgin curve is dependent upon the location of Point "P". In addition to the above, the slope of the recompression curve approximates the slope of the rebound curve only in instances when the rebound curve is very flat. This situation is generally encountered when the applied load is less than the preconsolidation pressure. Therefore, it is the writer's opinion that extreme care should be used when reconstructing the recompression portion of the true curve from laboratory rebound curves.

1. Engineer, Soils Mechanics, So. Atlantic Div. Laboratory Corps of Engineers, U. S. Army, Marietta, Georgia.

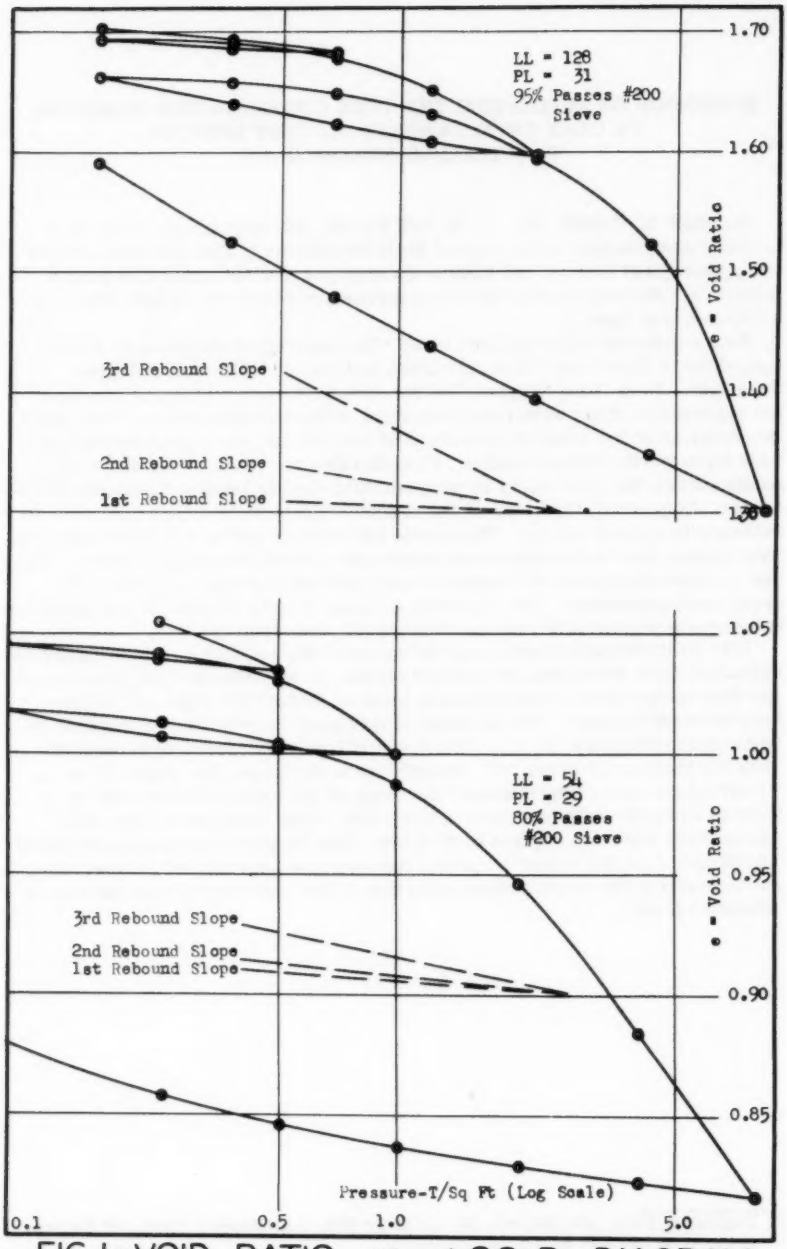


FIG 1—VOID RATIO vs LOG P DIAGRAM

DISCUSSION OF THE STRUCTURE OF INORGANIC SOIL
PROCEEDINGS-SEPARATE 315

E. S. BARBER,[1] A. M. ASCE.—While the surface characteristics of clay minerals are important to soil structure, grosser factors should not be ignored such as layers or lenses, varves, partings, shrinkage or shear cracks, root holes and differential weathering and seepage. Between the extremes of surface chemistry and macroscopic structure, the distribution of soil particles and moisture in apparently homogeneous soil masses have very marked effects. This discussion concerns Sections XVII, XVIII, and XIX.

Consider a sample of powdered soil packed dry and then saturated, compared to a second sample with the same final moisture and density prepared by consolidating a portion of the same soil material after mixing with water. The sample packed dry will have a non-uniform particle arrangement or structure, while the sample mixed with water will have a more uniform structure (See figure A). For the same overall density, the non-uniform structure produces greater permeability, less compressibility, and more swell or rebound when compressive loads or capillary tensions are released. Swelling of clay masses is caused by straightening of bent particles as when load is removed from an aggregation of mica flakes. For example, samples of a soil compacted to the same density at different moisture contents were all immersed; some were allowed to rebound under a light load immediately, while others were temporarily (24 hours) restrained from swelling externally. The dryer samples not only swelled most due to greater internal strain built-in during compaction, but also showed greater reduction in external swelling due to temporary restraint because the non-uniform structure permitted relief of stress by swelling into the less dense areas.[2] Similarly, for soil not so wet as to fail under the hammer, tamping gives more uniform structure than static compaction (confined in a cylinder between close fitting pistons) and therefore less permeability, more compressibility and less swelling.

Many effects of chemicals are due to their control of dispersion which determines the uniformity of structure if there is sufficient freedom of movement of particles. For example, exchange of bases after a soil was compacted changed the permeability but did not change the swelling characteristics until after the soil was mixed to the liquid limit.[2]

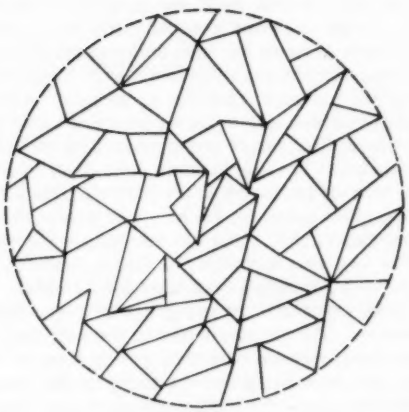
Consolidation tests on a powdered clay mixed with water, benzene, alcohol and acetone showed least permeability and greatest consolidation and swelling for soil mixed with water. This is attributed to the fact that water released the surface tension in the powdered clay thus giving a more uniform structure than the other liquids and permitting the elasticity of the platy clay particles to come into play. With benzene the highly stressed aggregations which make up the powdered clay were never dispersed.

Thixotropy can be explained as a weakening due to shear surfaces created between aggregations by disturbance. Thus remolding a uniform soil tends

1. Consulting Engineer, Arlington, Virginia.
2. "Structure of Disturbed Soils" by E. S. Barber, Proc. 2nd Int. Conf. on Soil Mechanics and Foundation Engineering, Rotterdam 1948, V. 5, p. 24.

to densify groups of particles while creating looser areas between them. Rest after disturbance gives a regain in strength as the densified groups resorb water from the looser areas thus approaching a more uniform arrangement of particles. The existence of a continuous framework of solid particles even in very wet soil was shown when sublimation of the solvent from a frozen gel of clay left an elastic sponge-like framework.[3]

NON-UNIFORM



UNIFORM

FIGURE A -- SOIL STRUCTURE AT SAME DENSITY

3. "Thixotropy of Clays," by R. Fahn, A. Weiss and U. Hoffmann, Ceramic Abstracts, Journal, American Ceramic Society, Oct. 1953, p. 183.

DISCUSSION OF THE STRUCTURE OF INORGANIC SOILS
PROCEEDINGS-SEPARATE 315

DR. I. TH. ROSENQUIST[1]—Dr. Lambe's way of looking upon the inorganic soils is very interesting. As he says, it is necessary to strengthen the frame work of soil mechanics; we have now to investigate the physico-chemical forces acting in the soils.

Research work, carried out by the Norwegian Geotechnical Institute during the last year has brought us to an opinion which is very close to that of Dr. Lambe's. We are now preparing a major paper, concerning the physical chemistry of Norwegian clays. The results obtained by us, seem to prove some points of Dr. Lambe's. Some of them give us a slightly different view so we think it desirable to present this discussion.

The Norwegian soils are mainly illitic whereas the soils treated by Dr. Lambe seem to be montomorilonitic and kaolinitic. This difference in mineral composition is, however, unimportant as to the general idea of the mechanical properties.

The most interesting point in Dr. Lambe's view is his statement that the cohesion is the result of forces acting between the particles. (Op. cit. 315, p. 32). These forces are regarded as mainly van der Waals forces. (Op. cit. 315, p. 31). The van der Walls forces are dependent not only upon the distances between the considered particles, but also upon the polarizability of the particles. The polarizability of an insulator is a function of the dielectric-constant and the volume. In clays, it is however not the volume of the clay particle itself which is of importance, but the volume of the adsorbed cation. We have carried out investigations in the strength of clays with different cations, using adsorbed lithium, sodium, potassium and caesium. The ions are of the following radii:

$Li^+ = 0.60$ Å, $Na^+ = 0.95$ Å, $K^+ = 1.33$ Å and $Cs^+ = 1.69$ Å.

The shear strength of a clay from Åsrum in the south-east of Norway was investigated in the remolded state at 2 different water contents, viz. 52% and 57%. We found the following values for the shear strength:

Lithium clay, 52% water: S = 0.17 tons per square meter
Lithium clay, 57% water: S = 0.11 tons per square meter

Sodium clay, 52% water: S = 0.25 tons per square meter
Sodium clay, 57% water: S = 0.15 tons per square meter

Potassium clay, 52% water: S = 0.50 tons per square meter
Potassium clay, 57% water: S = 0.30 tons per square meter

Caesium clay, 52% water: S = 0.91 tons per square meter
Caesium clay, 57% water: S = 0.67 tons per square meter

Several other experiments show the same order. In these cases we have dealt with neutralized clays. We regarded as neutralized those clays which contain sufficient electrolyte in the water phase to press the zeta potential to

1. Norwegian Geotechnical Institute of Oslo, Norway.

zero. This means that the amount of cations present within the rigid water film round the clay mineral is sufficient to neutralize the negative charges of the mineral grain. This again means that the repulsive forces, acting between the clay minerals, are at a minimum. By decreasing electrolyte content of the water phase, the zeta-potential will increase so that the diffused double layer will extend beyond the rigid water film. This gives an increased repulsion between the clay minerals.

The attractive forces are, however, very little influenced by this leaching process. The resulting forces, that is, the sum of the attractive and the repulsive forces, will however decrease. This is clearly demonstrated in an experiment carried out in our laboratory: "2 clay sediments were prepared and consolidated by 9 tons per square meter with a water phase containing 35 g sodium chloride per liter. Subsequently, under the load of consolidation, the sodium chloride was leached from the one sample and not from the other. The water content of the sediment was uninfluenced by the leaching.

The shear strength of the leached clay in the undistributed state dropped roughly to 60%, whereas the shear strength in remolded state dropped to about 1/1000 of the original strength. Thus the sensitivity became very large. The zeta-potential increased very much.

As the repulsive forces acting between clay minerals are mainly of electrostatic nature, the influence of such forces upon flaky minerals will tend to make them parallel to each other. Such an arrangement will take up less volume than the unoriented arrangement.

If clay is sedimented in waters, containing increasing amounts of electrolytes, the zeta potential and consequently the repulsive forces will decrease by increasing salt concentration in the water. This will lead to a steadily decreasing order of the clay particles of the sediment and thus to an increasing void ratio. The photographs No. 1 and 2 show the volume of 2 grams of two illitic clays sedimented in waters of different salinities. The photos were taken 6 and 10 weeks after the sedimentation and represent the final volume of the sediment as a function of the salinity. Picture No. 3 shows the volume of 2 grams of clay, sedimented in water containing sodium chloride, potassium chloride, and caesium chloride. In all these cases the salinity was so high that the zeta potential was at a minimum. We have therefore in these cases a picture of the attractive forces between the particles. These pictures illustrate the effect, shown in the two upper drawings in Dr. Lambe's paper, p. 38. Photo No. 4 shows a thin section of an undisturbed marine clay in polarized light at a magnification of 2700 times. It is seen that the flaky minerals are poorly oriented, whereas the larger grains have a certain degree of orientation.

As will be seen our view is not completely in agreement with Dr. Lambe's view as expressed on p. 315-46, where he states that: "The hydrated sodium ions, in effect, push the particles apart because they increase the thickness of the diffuse double layer and therefore the adsorbed water layer". It does not seem necessary that an increasing thickness of the diffuse double layers will increase the thickness of the adsorbed water layer, but it will increase the zeta-potential and the repulsive forces acting between the clay particles. A similar effect is obviously obtained when the polyphosphate is added to the clay (Dr. Lambe's paper, p. 46), because of the bonding between the negatively charged polyphosphate anion and the clay particles. Thus the total negative charge is increased, and as clearly shown by Dr. Lambe's experiment, the remolded strength is decreased because of the repulsion between the particles + the polyphosphate anion.

Oslo, 10th January 1954.

I. Th. Rosenquist

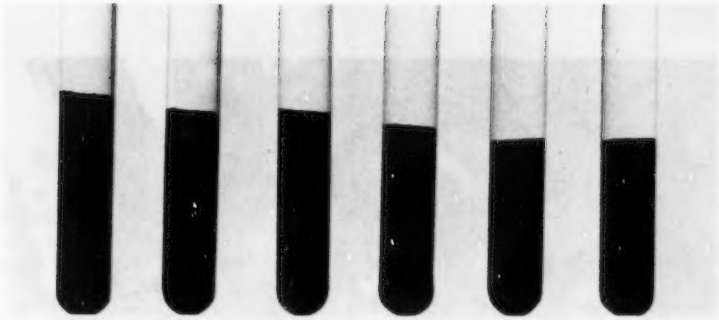


Fig. 1

Fig. 2

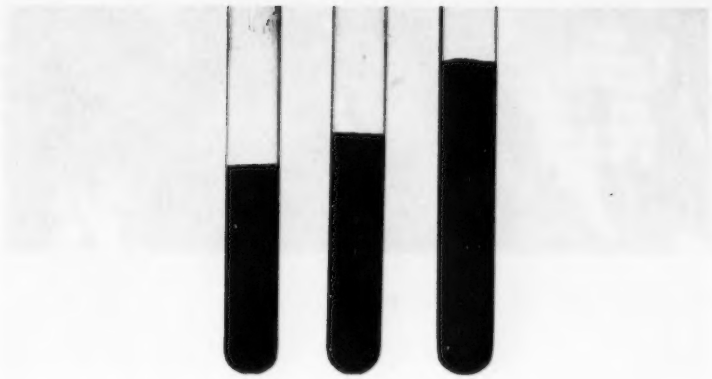


Fig. 3

Fig. 4

DESCRIPTION OF ILLUSTRATIONS

Fig. No. 1.

2 g illitic clay, sedimented in waters of increasing salinities. The clay material was taken from a natural deposit with a salt content of the pore water of 0.21% . The salt contents in the reagent tube is from right to left: 0% , 0.21 %, 0.23% , 0.26 %, 0.32 % and 0.44 %.

The photo was taken 6 weeks after sedimentation. There is now equilibrium and no further sinking. The strength of the sediment is just sufficient to carry its own weight.

Fig. No. 2.

2 g illitic clay, sedimented in waters of increasing salinities. The clay material for this series was taken from a natural deposit where the salinity was zero. The picture was taken 10 weeks after sedimentation and represents equilibrium in the same way as picture No. 1. The salt content of the water in the different tubes is from left to right: 0 %, 0.1% , 0.2 %, 0.4 %, 0.8 %, and 1.6% .

Fig. No. 3.

2 g illitic clay material, sedimented in water, containing from left to right 0.75 normal solution of sodium chloride, potassium chloride and caesium chloride. The picture was taken 10 weeks after sedimentation and represents equilibrium in the same way as picture No. 1 and No. 2.

Fig. No. 4.

A thin section of a marine post glacial clay from the Oslo region. The specimen is prepared by gradual replacement of the water phase by sulphonated fatty alcohol, which is a waxy substance. The replacement was performed shortly above the melting points of the detergent. Subsequently the specimen now impregnated was cooled down below the melting point of the detergent and the thin slice was prepared by means of a biological microtome. The photo is taken in polarized light, showing the haphazardly distributed flaky mineral grains, which forms the matrix between the larger minerals of silt and sand size.

PROCEEDINGS-SEPARATES

The technical papers published in the past twelve months are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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a. Beginning with "Proceedings-Separate No. 200," published in July, 1953, the papers were printed by the photo-offset method.
b. Presented at the Miami Beach (Fla.) Convention of the Society in June, 1953.
c. Presented at the New York (N.Y.) Convention of the Society in October, 1953.
d. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.
e. Discussion of several papers, grouped by divisions.
f. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.

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